Significant changes from the 2008 to the 2011 edition of ACI 318

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- This paper summarizes the significant changes made since the publication of the 2008 *Building Code Requirements for Structural Concrete (ACI 318-08) and Commentary (ACI 318R-08)* that are reflected in the 2011 edition of the code.
- Changes affecting conventionally reinforced concrete as well as precast, prestressed concrete, including posttensioned concrete, are enumerated.
- The changes to "Appendix D: Anchoring to Concrete," are particularly important and are of major interest to the precast/prestressed concrete industry. These are described in detail.

The American Concrete Institute (ACI) has published *Building Code Requirements for Structural Concrete (ACI 318-11) and Commentary (ACI 318R-11)*.[1] ACI 318-11 has been adopted by the 2012 *International Building Code* (IBC).[2] Thus, whenever the 2012 IBC is adopted by a local jurisdiction, as it will be by the State of California on January 1, 2014, ACI 318-11 will be law within that jurisdiction.

Although the changes from ACI 318-08[3] to ACI 318-11 are not as extensive or as substantive as those from ACI 318-05[4] to ACI 318-08, some of the changes in the latest cycle have significant effects on the design and construction of concrete structures.

Chapter 1: General Requirements

In section 1.1.4, ACI 332-04 *Residential Code Requirements for Structural Concrete*[5] has been updated to ACI 332-10.[6]

In commentary sections R1.1.8.1 and R.1.1.8.2, two standards published by the Steel Deck Institute (SDI) are referenced: *Standard for Non-Composite Steel Floor Deck* (ANSI/SDI NC-2010)[7] and *Standard for Composite Steel Floor Deck* (ANSI/SDI C1.0-2006).[8] The first document refers to ACI 318 for the design and construction of the structural concrete slab. The second document refers to the appropriate portions of ACI 318 for the design and construction of the concrete portion of the composite assembly. *Design Manual for Composite Decks, Form Decks, and Roof Decks*,[9] published by SDI, is also referenced. ACI 318 previously referenced *Standard for the Structural Design of Composite Slabs* (ANSI/ASCE 3)[10] for the design of composite slabs and *Standard Practice for the Construction and Inspection of Composite Slabs* (ANSI/ASCE 9)[11] for guidelines on the construction of composite steel deck slabs.

In commentary section R1.1.9.1, the references have been updated from the 2005[12] to the 2010[13] ASCE 7/SEI standard *Minimum Design Loads for Buildings and Other Structures*, from the 2006[14] to the 2009[15] edition of the *International Building Code*, and from the 2006[16] to the 2009[17] edition of the National Fire Protection Association (NFPA) 5000 *Building Construction and Safety Code*. These newer editions have been added to Table R1.1.9.1, Correlation between Seismic-Related Terminology in Model Codes. The following sentence has been added at the end of commentary section R1.1.9.1: "The model building codes also specify overstrength factors, Ω_0, that are related to the seismic-force-resisting system used for the structure and used for the design of certain elements."

Section 1.1.10 states that ACI 318 does not govern the design and construction of tanks and reservoirs. Section R1.1.10 now tells the user that guidance for the design and construction of cooling towers and circular prestressed concrete tanks is found in the reports of ACI committees 334 Concrete Shell Design and Construction,[18] 350 Environmental Engineering Concrete Structures,[19] 372 Tanks Wrapped with Wire or Strand,[20] and 373 Tanks with Internal Tendons.[21] This is an expanded version of commentary section R19.1.1 of ACI 318-08, which has been moved to chapter 1 of ACI 318-11.

Section 1.2 now requires "Type, size, and location of anchors, and anchor installation and qualification requirements in accordance with D.9" to be shown in contract documents.

Chapter 2: Notations and Definitions

The definition for headed deformed bars in ACI 318-08 contained a number of requirements for the head. The definition now refers to section 3.5.9, which in turn references Annex A1 Requirements for Class HA Head Dimensions of ASTM A970 *Standard Specification for Headed Steel Bars for Concrete Reinforcement*.[22]

Definitions have been added for vertical wall segment and wall pier.

ASTM A82 *Standard Specification for Steel Wire, Plain, for Concrete Reinforcement,*[23] ASTM A185 *Standard Specification for Steel Welded Wire Reinforcement, Plain, for Concrete,*[24] ASTM A496 *Standard Specification for Steel Wire, Deformed, for Concrete Reinforcement,*[25] and ASTM A497 *Standard Specification for Steel Welded Wire Reinforcement, Deformed, for Concrete*[26] have been combined into ASTM A1064 *Standard Specification for Steel Wire and Welded Wire Reinforcement, Plain and Deformed, for Concrete.*[27] This change is reflected in the definition of welded wire reinforcement in section 2.2 of ACI 318-11.

Chapter 3: Materials

Section 3.2.1 now refers to "slag cement," rather than "ground-granulated blast-furnace slag," because ASTM has changed the title of ASTM C989 to *Standard Specification for Slag Cement for Use in Concrete and Mortars*.[28]

ASTM A615 *Standard Specification for Deformed and Plain Carbon Steel Bars for Concrete Reinforcement*[29] and ASTM A706 *Standard Specification for Low-Alloy Steel Deformed and Plain Bars for Concrete Reinforcement*[30] (section 3.5.3.1) have both added Grade 80 reinforcement, which has a minimum yield strength of 80,000 psi (550 MPa). The use of this reinforcement is not permitted by section 21.1.5 in special moment frames and special structural walls. Available data were judged to be insufficient to confirm applicability of existing code provisions to special moment frames and special structural walls reinforced with steel having yield strength higher than 60,000 psi (410 MPa).

Section 3.5.3.2 of ACI 318-08 required that "for bars with f_y exceeding 60,000 psi, the yield strength shall be taken as the stress corresponding to a strain of 0.35 percent." ACI defines f_y as specified yield strength of reinforcement. The same section in ACI 318-11 requires that "for bars with f_y less than 60,000 psi, the yield strength shall be taken as the stress corresponding to a strain of 0.5 percent, and for bars with f_y at least 60,000 psi, the yield strength shall be taken as the stress corresponding to a strain of 0.35 percent." This definition of yield strength overrides the one prescribed in ASTM A615, A706, A995, and A996.

Section 3.5.3.8 permits the use of zinc and epoxy dual-coated reinforcing bars conforming to ASTM A1055 *Standard Specification for Zinc and Epoxy Dual-Coated Steel Reinforcing Bars*.[31]

Section 3.5.9 now requires ASTM A970 headed deformed bars to conform to Annex A1 Requirements for Class HA Head Dimensions. The commentary explains that the limitation to Class HA head dimensions from Annex A1 of ASTM A970 is due to a lack of test data for headed deformed bars that do not meet Class HA dimensional requirements. While ACI 318-11 references ASTM A970-09,[22] ACI 318-08 referenced ASTM A970-06,[32] which did not have an Annex A1. ACI 318 required that "obstructions and interruptions of the bar deformations, if any, shall not extend more than $2d_b$ from the bearing face of the head" (d_b is the nominal diameter of bar).

Chapter 4: Durability Requirements

It is required in Table 4.2.1 that percent sulfate by mass in soil be determined by ASTM C1580 *Standard Test for Water-Soluble Sulfate in Soil*[33] and that concentration of dissolved sulfates in water in parts per million (ppm) be determined by ASTM D516 *Standard Test Method for Sulfate Ion in Water*[34] or ASTM D4130 *Standard Test Method for Sulfate Ion in Brackish Water, Seawater, and Brine.*[35]

Section R4.5.1 says that ACI 222R-01 *Protection of Metals in Concrete against Corrosion*[36] has adopted chloride limits, test methods, and construction types and conditions that are slightly different from those in ACI 318, as shown in Table R4.3.1. It also says that ACI 201.2R-08 *Guide to Durable Concrete*[37] has adopted these same limits by referring to ACI 222R-01.

Chapter 5: Concrete Quality, Mixing, and Placing

For the purpose of establishing standard deviation for test records, a test record obtained less than 12 months before a submittal was acceptable under ACI 318-08. The 12-month limit has now been extended to 24 months in ACI 318-11 section 5.3.1.1.

ACI 318-08 required documentation showing that proposed concrete mixture proportions will produce an average compressive strength equal to or greater than the required average compressive strength to consist of one or more field strength test record(s) or trial mixtures not more than 12 months old. The 12-month limit has now been extended to 24 months in ACI 318-11 section 5.3.3.

Section 5.6.1 now requires the testing agency performing acceptance testing of concrete to have minimum proficiency in compliance with ASTM C1077 *Standard Practice for Laboratories Testing Concrete and Concrete Aggregates for Use in Construction and Criteria for Laboratory Evaluation*.[38] Also, all reports of acceptance tests are required to be provided to the licensed design professional, contractor, concrete producer, and when requested, to the owner and the building official.

Commentary section R5.6.5 now clarifies that the instructions for investigation of low-strength test results are applicable only for evaluation of in-place strength at the time of construction. Strength evaluation of existing structures is covered in chapter 20.

Chapter 6: Formwork, Embedments, and Construction Joints

"Design drawings and specifications" has been changed to "contract documents" in sections 6.1.1 and 6.4.7. Other than that, there are no changes in this chapter.

Chapter 7: Details of Reinforcement

In section 7.7.6, which addresses corrosive environments and other severe exposure, "amount of concrete protection shall be suitably increased" has been changed to "the concrete cover shall be increased as deemed necessary and specified by the licensed design professional."

In section 7.10.4.5, which is about splicing of spiral reinforcement, the use of deformed zinc-coated (galvanized) bars, plain zinc-coated (galvanized) bars, and zinc-and-epoxy dual-coated deformed bars as spiral reinforcement is now recognized.

A new section 7.10.5.4 has been added, and it reads: "Where longitudinal bars are located around the perimeter of a circle, a complete circular tie shall be permitted. The ends of the circular tie shall overlap by not less than 6 in. [150 mm] and terminate with standard hooks that engage a longitudinal column bar. Overlaps at ends of adjacent circular ties shall be staggered around the perimeter enclosing the longitudinal bars." **Figure 1** illustrates the requirement.

In sections 7.12.3.2 through 7.12.3.5, new requirements have been added concerning temperature and shrinkage reinforcement in posttensioned slabs. These requirements define the gross area of beam and slab sections to be used for determining the effective prestress. A figure has been added to the commentary to better illustrate the intentions of the provision. **Figure 2** is an adaptation of the commentary figure. The primary reason for this code change was to clearly discourage the practice of providing all of the required shrinkage and temperature reinforcement in the beam web with none in the slab between beams.

Chapter 8: Analysis and Design—General Considerations

There are no changes in this chapter.

Chapter 9: Strength and Serviceability Requirements

The design load combinations in section 9.2 have been revised to be fully consistent with those of ASCE/SEI 7-10.[13] That standard has converted wind loads to strength level and changed the wind load factor in strength design from 1.6 to 1.0.

The less common loads—self-straining loads T, fluid pressure F, and horizontal earth pressure H—have been removed from the basic load combinations. They are now covered in sections 9.2.3, 9.2.4, and 9.2.5, respectively.

Chapter 10: Flexure and Axial Loads

"Lateral buckling shall be considered" has been deleted from section 10.7.1 because it is not a meaningful or enforceable requirement. Also, section 10.7.4 of ACI 318-08 has been deleted because it contained shear reinforcement requirements in a chapter devoted to flexure and axial loads.

Commentary section R10.10.2 has added the following text: "Several methods have been developed to evaluate slenderness effects in compression members that are subject to biaxial bending. A review of some of these methods is presented in Reference 10.34."[39]

Chapter 11: Shear and Torsion

Section 11.7 on deep beams has undergone several changes. Section 11.7.2 now reads: "Deep beams shall be designed either by taking into account nonlinear distribution of strain or by appendix A. In all cases, minimum distribution reinforcement shall be provided in accordance with 11.7.4." The first sentence is rewritten for clarity. The second sentence is an addition. Section 11.7.3 used to require V_n not to exceed $10\sqrt{f_c'}b_w d$, where V_n is nominal shear strength, f_c' is specified compressive strength of concrete, b_w is web width, and d is distance from extreme compression fiber to centroid of longitudinal tension reinforcement. It now requires V_u to be less than or equal to $\phi 10\sqrt{f_c'}b_w d$, where V_u is factored shear force at section and ϕ is strength reduction factor. Section 11.7.4, requiring distributed reinforcement along the sides of deep beams to be not less than that required in 11.7.4.1 and 11.7.4.2 is

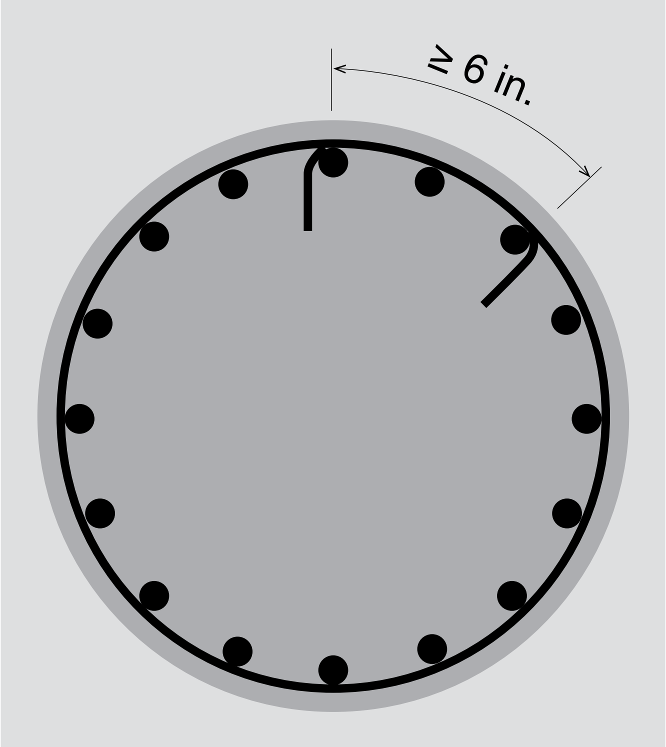

Figure 1. Circular tie configuration per section 7.10.5.4. Note: 1 in. = 25.4 mm.

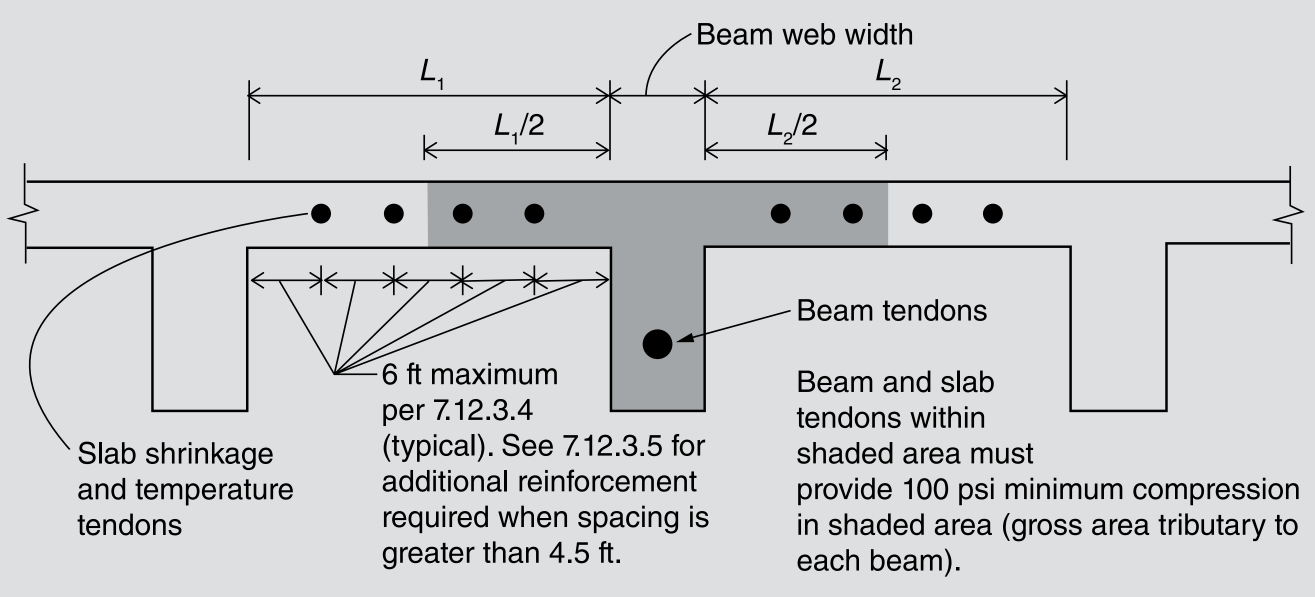

Figure 2. Gross area for determining effective prestress. Source: Adapted by permission from ACI, *Building Code Requirements for Structural Concrete (ACI 318-11) and Commentary (ACI 318R-11)* (2011), Fig. R7.12.3(a). Note: L_1 = clear slab span on left side of beam; L_2 = clear slab span on right side of beam. 1 in. = 25.4 mm; 1 ft = 0.305 m.

new. In section 11.7.4.2, the area of shear reinforcement parallel to the longitudinal axis of the beam is now required to be not less than $0.0025b_w s_2$, where s_2 is the center-to-center spacing of the longitudinal shear reinforcement. The 0.0025 was 0.0015 in ACI 318-08. The former section 11.7.6, which permitted provision of reinforcement satisfying A.3.3 instead of the minimum horizontal and vertical reinforcement specified in 11.7.4 (now 11.7.4.1) and 11.7.5 (now 11.7.4.2) has been deleted.

Commentary section R11.7 has been rewritten to reflect these changes and to explain some of them. The last sentence in section R11.7.4 now reads: "Tests … have shown that vertical shear reinforcement (perpendicular to the longitudinal axis of the member) is more effective for member strength than horizontal shear reinforcement (parallel to the longitudinal axis of the member) in a deep beam, but the specified minimum reinforcement in both directions is required to control the growth and width of diagonal cracks." This explains the increase in the amount of the minimum horizontal shear reinforcement as well as the deletion of former section 11.7.6.

Chapter 12: Development and Splices of Reinforcement

The factor used to modify development length based on reinforcement coating Ψ_e given in section 12.2.4(b), applicable in ACI 318-08 to epoxy-coated bars and wires, has now been made applicable to zinc-and-epoxy dual-coated bars.

Part of commentary section R12.6 Development of Headed and Mechanically Anchored Deformed Bars in Tension has been rewritten to reflect the change discussed under chapter 3, item 5.

In 2011, the excess reinforcement factor for headed bars in section 12.6.2 was removed from the code. The excess reinforcement factor $A_{s\ required}/A_{s\ provided}$, (where $A_{s\ required}$ is area of nonprestressed longitudinal tension reinforcement required and $A_{s\ provided}$ is area of nonprestressed longitudinal tension reinforcement provided) applicable to deformed bars without heads, is not applicable for headed bars where force is transferred through a combination of bearing at the head and bond along the bar.

Chapter 13: Two-Way Slab Systems

In slabs with shear heads and in lift-slab construction, structural integrity reinforcement is now required to have Class B, rather than Class A, tension lap splices or mechanical or welded splices satisfying section 12.14.3.

Chapter 14: Walls

Commentary section R14.8.4 references ASCE 7 Appendix C: Serviceability Considerations. The text has been updated to be consistent with ASCE/SEI 7-10.

Chapter 15: Footings

There are no changes in this chapter.

Chapter 16: Precast Concrete

There are only minor revisions to this chapter.

Chapter 17: Composite Concrete Flexural Members

There are no changes in this chapter.

Chapter 18: Prestressed Concrete

The permissible stress of $0.82f_{py}$ (where f_{py} is specified yield strength of prestressing steel) but not greater than $0.74f_{pu}$ (where f_{pu} is specified tensile strength of prestressing steel) in prestressing steel immediately upon prestress transfer in section 18.5.1 has been eliminated based on practical experience with posttensioned concrete members. Commentary section R18.5.1 is now considerably shorter and much more direct.

The formulas for estimating friction loss in posttensioning tendons have been eliminated from section 18.6.2.1 as being textbook material. That section now simply states: "The required effective prestress force shall be indicated in the contract documents." Table R18.6.2, giving friction coefficients for posttensioning tendons for use in the deleted formulas, has also been eliminated. Section 18.6.2.2 now reads: "Computed friction loss shall be based on experimentally determined wobble and curvature friction coefficients." Section 18.6.2.3 says: "The prestress force and friction losses shall be verified during tendon stressing operations as specified in 18.20."

Commentary section R18.7.2 has been expanded to provide guidance on the value of γ_p for various types of prestressing reinforcement. The γ_p term in Eq. (18-1) reflects the influence of the stress-strain properties of different types of prestressing reinforcement on the value of f_{ps}, stress in prestressing steel at nominal flexural strength.

Commentary section R18.9.3.2 now clarifies how to compute the minimum bonded reinforcement corresponding to resultant tensile force N_c in positive moment areas. In chapter 2, the definition of N_c now makes it clear that it includes the combined effects of all service loads and effective prestress.

Chapter 19: Shells and Folded Plate Members

There are no changes in this chapter.

Chapter 20: Strength Evaluation of Existing Structures

There are no changes in this chapter.

Chapter 21: Earthquake-Resistant Structures

In sections 21.1.4.1 and 21.1.5.1, references to "special structural walls and coupling beams" have now been changed to "special structural walls, and all components of special structural walls including coupling beams and wall piers." This is in view of the inclusion of wall pier provisions in section 21.9 of ACI 318-11.

ACI 318-08 section 21.1.5.2 required deformed reinforcement resisting earthquake-induced flexure, axial force, or both to comply with ASTM A706,[40] except that ASTM A615[41] grades 40 and 60 reinforcement were permitted subject to two supplementary requirements. ACI 318-11 requires the ASTM A706 reinforcement to be Grade 60. This is in order to exclude the new Grade 80 reinforcement that has been added to ASTM A706.

Section 21.3.3 of ACI 318-08 provided two choices for the calculation of the required shear strength of a column of an intermediate moment frame. It could be calculated as the sum of the shear associated with the development of nominal moment strength at each restrained end of the clear span and the shear calculated for factored gravity loads. Alternatively, it could be calculated as the maximum shear obtained from design load combinations that include E (where E is effects of earthquake or related internal moments and forces), with E assumed to be twice that prescribed by the legally adopted general building code for earthquake-resistant design. In the new section 21.3.3.2 of ACI 318-11, the multiplier of two has been increased to the overstrength factor of the intermediate moment frame Ω_0, which is three. The multiplier of two was determined to be unconservative.

In ACI 318-08 section 21.5.3.2, the spacing of hoops within the region of potential plastic hinging at each end could not exceed the smallest of the following:

- $d/4$
- 8 times the diameter of the smallest longitudinal bars
- 24 times the diameter of the hoop bars
- 12 in. (300 mm)

In ACI 318-11 section 21.5.3.2, item (b) has been changed to six times the diameter of the smallest primary flexural reinforcing bars, excluding longitudinal skin reinforcement required by section 10.6.7. Item (c) has been deleted. Item (d) now is 6 in. (150 mm). For deeper beams, this is a significant decrease in the spacing of confinement reinforcement in the regions of potential plastic hinging. It is intended to improve confinement in these regions.

Section 21.5.3.3 has been expanded to read as follows: "Where hoops are required, primary flexural reinforcing bars closest to the tension and compression faces shall have lateral support conforming to 7.10.5.3 or 7.10.5.4. The spacing of transversely supported flexural reinforcing bars shall not exceed 14 in. [360 mm]. Skin reinforcement required by 10.6.7 need not be laterally supported."

A new section 21.6.3.2 has been added, requiring that in columns with circular hoops, the minimum number of longitudinal bars be six.

For a special shear wall for which special boundary elements were required, section 21.9.6.4(e) stated: "Horizontal reinforcement in the wall web shall be anchored to develop f_y within the confined core of the boundary element." The requirement has now been expanded as follows: "Horizontal reinforcement in the wall web shall extend to within 6 in. [150 mm] of the end of the wall. Reinforcement shall be anchored to develop f_y in tension using standard hooks or heads. Where the confined boundary element has sufficient length to develop the horizontal web reinforcement and $A_v f_y / s$ (where A_v is area of shear reinforcement within spacing s, and s is center-to-center spacing of shear reinforcement) of the web reinforcement is not greater than $A_{sh} f_{yt} / s$ (where A_{sh} is total cross-sectional area of transverse reinforcement (including crossties) within spacing s and perpendicular to dimension b_c, f_{yt} is the specified yield strength of transverse reinforcement, and b_c is cross-sectional dimension of member core measured to the outside edges of the transverse reinforcement composing area A_{sh}) of the boundary element transverse reinforcement parallel to the web reinforcement, it shall be permitted to terminate the web reinforcement without a standard hook or head." **Figure 3** illustrates this.

Door and window openings in shear walls often lead to narrow vertical wall segments, many of which have been defined as wall piers in the IBC[2] and in the *Uniform Building Code* (UBC)[42] before it. Wall pier provisions are now included for the first time in the new section 21.9.8 of ACI 318-11. The dimensions defining wall piers are given in section 2.2.

Shear failures of wall piers have been observed in previous earthquakes. The intent of section 21.9.8 is to prescribe detailing that would result in sufficient shear strength of wall piers so that failure will be flexure governed, rather

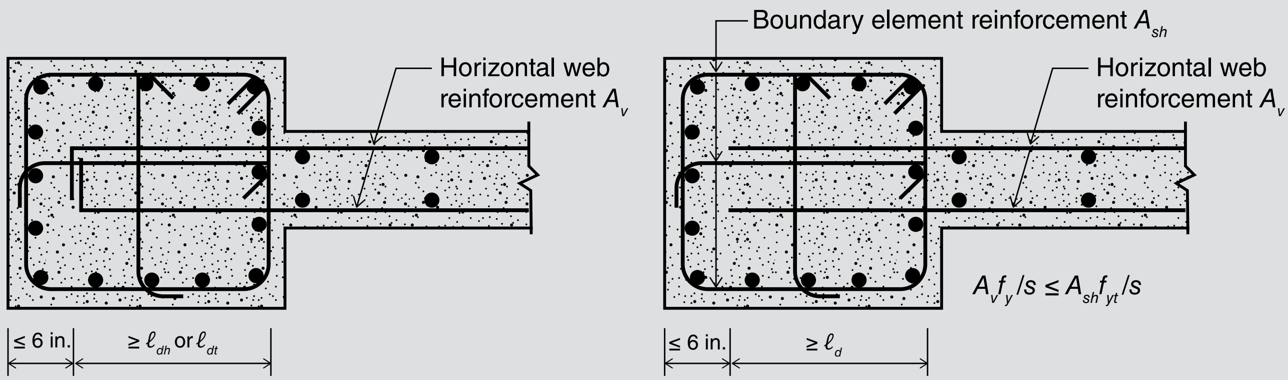

Figure 3. Development of wall horizontal reinforcement in confined boundary element. Note: f_y = specified yield strength of reinforcement; f_{yt} = specified yield strength of transverse reinforcement; ℓ_d = development length in tension of deformed bar, deformed wire, plain and deformed welded wire reinforcement, or pretensioned strand; ℓ_{dh} = development length in tension of deformed bar or deformed wire with a standard hook, measured from critical section to outside end of hook (straight embedment length between critical section and start of hook [point of tangency] plus inside radius of bend and one bar diameter); ℓ_{dt} = development length in tension of headed deformed bar, measured from the critical section to the bearing face of the head; s = center-to-center spacing of shear reinforcement. 1 in. = 25.4 mm.

than shear governed. The provisions apply to wall piers considered part of the seismic force–resisting system. Provisions for wall piers not considered part of the seismic force–resisting system are given in section 21.13.

Wall piers having $(\ell_w/b_w) \le 2.5$ (where ℓ_w is the length of the entire wall or wall segment or wall pier considered in the direction of the shear force) behave essentially as columns. Section 21.9.8.1 requires them to be detailed like columns. Alternative requirements are provided for wall piers having $(\ell_w/b_w) > 2.5$. The design shear force determined according to section 21.6.5.1 may be unrealistically large in some cases. As an alternative, section 21.9.8.1(a) permits the design shear force to be determined using load combinations in which the earthquake load effect has been amplified to account for member overstrength.

Wall piers at the edge of a wall are addressed in section 21.9.8.2. Under in-plane shear, inclined cracks can propagate into segments of the wall directly above and below the wall pier. Shear failure within the adjacent wall segments can occur unless sufficient reinforcement is provided in the adjacent wall segments (Fig. R21.9.8).

A new Table R21.9.1 in the commentary effectively summarizes the new requirements.

Commentary section R21.10.3 has been expanded to reference ACI *Requirements for Design of a Special Unbonded Post-Tensioned Precast Shear Wall Satisfying ACI ITG-5.1 and Commentary* (ACI ITG-5.2-09),[43] which defines design requirements for one type of special structural walls constructed using precast concrete and unbonded posttensioned tendons.

Chapter 22: Structural Plain Concrete

A new section 22.2.4 has been added, requiring that modification factor for lightweight concrete λ in chapter 22 be in accordance with section 8.6.1.

Appendix A: Strut-and-Tie Models

There are no changes in this appendix.

Appendix B: Alternative Provisions for Reinforced and Prestressed Concrete Flexural and Compression Members

There are no changes in this appendix.

Appendix C: Alternative Load and Strength Reduction Factors

The alternative strength design load combinations in section C.9.2.2 have been revised to be fully consistent with those of ASCE/SEI 7-10.[13] That standard has converted wind loads to strength level and changed the wind load factor in strength design to 1.0.

Appendix D: Anchoring to Concrete

The onerous nature of seismic design imposed by ACI 318-08 section D.3.3 on anchors in Seismic Design Category (SDC) C or higher is alleviated and the seismic design of anchors is made considerably more reasonable. Where the tension component of the strength-level earthquake force applied to the anchor or group of anchors is equal to or less than 20% of the total factored anchor tensile force determined from the same load combination, the seismic design requirements of section D.3.3 to prevent a brittle tension failure of anchors simply do not apply anymore (section D.3.3.4.1). Similarly, where the shear component of the strength-level earthquake force applied to the anchor or group of anchors is equal to or less than 20% of the total factored anchor shear force determined from the same load combination, the seismic design requirements of section D.3.3 to prevent a brittle shear failure of anchors simply do not apply anymore (section D.3.3.5.1).

Where the seismic component of the total factored tension demand on an anchor or a group of anchors exceeds 20%, the following four options have been made available:

a. Ensure failure of ductile steel anchor ahead of the brittle failure of concrete (section D.3.3.4.3(a)). In other words, the strength of ductile steel anchors needs to be smaller than the strengths calculated from various concrete failure modes. In ACI 318-08, this check was based on the design strengths of anchors determined from the considerations of steel anchor failure and concrete failure under tension. In ACI 318-11, this check is made less onerous in two ways:

    - The ductility check is to be performed now based on the nominal strengths associated with ductile steel anchor and concrete failure modes. This is easier to satisfy than a check based on design strengths because the ϕ-factors applied to concrete failure modes are smaller than that applied to steel anchor failure.

    - In ACI 318-08, the concrete failure strengths were reduced by a factor of 0.75. In ACI 318-11, for the purpose of this ductility check, the 0.75 factor is replaced by a factor of 1.2 on the steel strength. This is equivalent to applying a factor of $^{1}/_{1.2}$ = 0.83 on the concrete strengths, an 11% increase from before.

    In addition, this ductility check now involves the new concept of a *stretch length*: a minimum unbonded length of 8 times the diameter of the anchor to ensure an adequate ductile rotational capacity of the connection for proper energy dissipation. The stretch length can be provided outside of concrete by using an anchor chair (**Fig. 4**) or by debonding part of the anchor within concrete.

b. Design the anchor for the maximum tension force that can be transmitted by a ductile metal attachment after considering the overstrength and strain hardening of the attachment (section D.3.3.4.3(b)).

c. Design for the maximum tension force that can be transmitted by a nonyielding attachment (section D.3.3.4.3(c)).

d. Design for the maximum tension force obtained from design load combinations involving E, with E increased by Ω_0 (section D.3.3.4.3(d)).

For an anchor or a group of anchors subject to shear, three options similar to b, c, and d have been made available. Unlike ACI 318-08, ductile anchor failure in shear is not an option anymore.

As in ACI 318-08, in calculation of the design strength of an anchor or a group of anchors subject to the seismic design requirements for tension, concrete-governed strength is multiplied by a factor of 0.75, while steel-governed strength is not (section D.3.3.4.4). However, for anchors subject to the seismic design requirements for shear, ACI 318-11 does not impose this 0.75 factor on the concrete-governed strengths anymore.

In ACI 318-08 and earlier editions, the steel strength and pullout strength of anchors in tension and the steel strength in shear of a group of anchors were calculated based on the strength of a single anchor multiplied by the number of anchors in the group. Unless the anchors are all loaded equally, this can lead to a situation where the most highly stressed anchor could fail before reaching the calculated capacity of the group. In ACI 318-11, Table D.4.1.1 prescribes how to compute the strength of an anchor group depending on the failure mode and based on the most highly stressed anchor.

The maximum anchor diameter for which the provisions of sections D.5.2 and D.6.2 can be applied to calculate the concrete breakout strength in tension and shear, respectively, has been increased from 2 to 4 in. (50 to 100 mm) (section D.4.2.2). This expansion is based on the results from new tests conducted using larger-diameter anchors. However, a new Eq. (D-34) has also been introduced for an upper-bound value of basic concrete breakout strength in shear for a single anchor V_b to account for the larger-diameter anchors.

ACI 318-08 also imposed a 25 in. (635 mm) limitation on the anchor embedment depth for the calculation of concrete breakout strength using the provisions of appendix D. This limitation was effectively removed by section 1908.1.10 of the 2009 IBC.[15] ACI 318-11 does not have this limitation anymore.

An adhesive anchor is defined in section D.1 as "a post-installed anchor, inserted into hardened concrete with an anchor hole diameter not greater than 1.5 times the anchor diameter, that transfers loads to the concrete by bond between the anchor and the adhesive, and bond between

Figure 4. Use of anchor chair for providing stretch length. Photo courtesy of J. Silva, Hilti North America.

the adhesive and the concrete" (**Fig. 5**). The method of calculating nominal strength of adhesive anchors in bond failure is provided, along with requirements for testing and evaluation of adhesive anchors for use in cracked concrete or subject to sustained loads. Failure modes postulated for other anchors apply to adhesive anchors as well, except that the calculation of strength in anchor pullout is replaced by the evaluation of adhesive bond strength in accordance with section D.5.5. The provisions for adhesive anchors include criteria for overhead anchors, seismic design requirements, installation and inspection requirements, and certification of adhesive anchor installers. Separately, a certification program has been established jointly by ACI and the Concrete Reinforcing Steel Institute. Characteristic bond stress of adhesive anchors depends on the installation method and use conditions anticipated during construction and during the anchor service life and can be obtained for cracked and uncracked concrete from tests performed and evaluated in accordance with ACI 355.4-11, *Acceptance Criteria for Qualification of Post-Installed Adhesive Anchors in* Concrete.[44] Alternatively, the minimum values given in Table D.5.5.2 can be used, provided the conditions outlined in section D.5.5.2 and in Table D.5.5.2 are satisfied.

Miscellaneous items

The term "design drawings and specifications" has been replaced with "contract documents" throughout ACI 318-11. "Lateral reinforcement" and "lateral ties" have been replaced with "transverse reinforcement" and "transverse ties," respectively.

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Figure 5. Adhesive anchor and bond failure of adhesive anchor. Photo courtesy of Rolf Eligehausen, University of Stuttgart.

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Notation

$A_{s\,provided}$ = area of nonprestressed longitudinal tension reinforcement provided

$A_{s\,required}$ = area of nonprestressed longitudinal tension reinforcement required

A_{sh} = total cross-sectional area of transverse reinforcement (including crossties) within spacing s and perpendicular to dimension b_c

A_v = area of shear reinforcement within spacing s

b_c = cross-sectional dimension of member core measured to the outside edges of the transverse reinforcement composing area A_{sh}

b_w = web width

d = distance from extreme compression fiber to centroid of longitudinal tension reinforcement

d_b = nominal diameter of bar

E = effects of earthquake or related internal moments and forces

f_c' = specified compressive strength of concrete

f_{ps} = stress in prestressing steel at nominal flexural strength

f_{pu} = specified tensile strength of prestressing steel

f_{py} = specified yield strength of prestressing steel

f_y = specified yield strength of reinforcement

f_{yt} = specified yield strength of transverse reinforcement

F = fluid pressure

H = horizontal earth pressure

ℓ_w = length of entire wall or length of wall segment or wall pier considered in direction of shear force

ℓ_d = development length in tension of deformed bar, deformed wire, plain and deformed welded wire reinforcement, or pretensioned strand

ℓ_{dh} = development length in tension of deformed bar or deformed wire with a standard hook, measured from critical section to outside end of hook (straight embedment length between critical section and start of hook [point of tangency] plus inside radius of bend and one bar diameter)

ℓ_{dt} = development length in tension of headed deformed bar, measured from the critical section to the bearing face of the head

L_1 = clear slab span on left side of beam

L_2 = clear slab span on right side of beam

N_c = resultant tensile force in positive moment

s = center-to-center spacing of shear reinforcement

s_2 = center-to-center spacing of longitudinal shear reinforcement

T = self-straining loads

V_b = concrete breakout strength in shear for a single anchor

V_n = nominal shear strength

V_u = factored shear force at section

γ_p = influence of different types of prestressing reinforcement on the value of f_{ps}

λ = modification factor for lightweight concrete

$ø$ = strength reduction factor

Ψ_e = factor used to modify development length based on reinforcement coating

Ω_0 = overstrength factor

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Abstract

Significant changes made since the publication of the 2008 *Building Code Requirements for Structural Concrete (ACI 318-08) and Commentary (ACI 318R-08)* that are reflected in the 2011 edition of the code are summarized. Changes affecting conventionally reinforced concrete as well as precast, prestressed concrete, including posttensioned concrete, are enumerated. The changes to “Appendix D: Anchoring to Concrete” are particularly important and are of major interest to the precast/prestressed concrete industry.

Keywords

ACI 318, code, structural concrete.

Review policy

This paper was reviewed in accordance with the Precast/Prestressed Concrete Institute’s peer-review process.

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